E/CONF.39/A/39
29 Sept.1962
ORIGINAL : ENGLISH

UNITED NATIONS CONFERENCE
ON THE APPLICATION OF SCIENCE AND TECHNOLOGY
FOR THE BENEFIT OF THE LESS DEVELOPED AREAS

Agenda Item: A.3.2.2

South Africa

SOME INNOVATIONS TO IMPROVE ESTIMATION OF RIVER FLOW AND DATA COLLECTION

By J.P. KRIEL,
J.S. WHITMORE,
K.J. HARVEY,
A.L. RABIE,
R. DE VILLIERS,
D.F. ROBERTS,

edited by

J.P. KRIEL
Hydrology Division,
Department of Water Affairs,
Republic of South Africa

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SOME INNOVATIONS TO IMPROVE ESTIMATION OF RIVER FLOW AND DATA COLLECTION. A.3.2.2.

By J.P. Kriel, J.S. Whitmore, K.J. Harvey, A.L.Rabie, R.de Villiers and D.F. Roberts. Edited by J.P. Kriel.
HYDROLOGY DIVISION, DEPARTMENT OF WATER AFFAIRS, REPUBLIC OF SOUTH AFRICA.

1. Where stream flow records are inadequate estimates often have to be made to assess water resources. This paper deals with some innovations used in the Republic of South Africa to improve the accuracy of data collection as well as the reliability of such estimates.

2. ESTIMATION OF RAINFALL OVER A CATCHMENT (J.S.Whitmore).

a. Inadequate streamflow records can in many cases be supplemented by estimates of flow based on rainfall, but even this solution may be complicated by a dearth of rain gauges in the catchment.

b. Improved methods have been devised for estimating mean rainfall over a catchment in which there are at least some gauges (1). These include a group mean, weighted for the area of zones obtained by a primary altitudinal subdivision of a catchment and a secondary subdivision based on aspect. In another method the area of influence of each station is represented on a map by a polygon based not on the distance between adjacent stations but on the projected altitudinal difference between them. In a third method, plotted stations are linked to form triangles whose area can be calculated from measurements of length, obviating the need for planimetry.

c. Multiple curvilinear regressions can be used profitably to establish the relationship between the mean annual rainfall of stations distributed over a broad area and their positional factors, so that these latter can be used to estimate the mean annual rainfall of any ungauged catchment within the area (2). The linear multiple regression is first determined, whereupon net linear regressions of mean annual rainfall (Y) of each of the independent variates (such as altitude, aspect and continentality) are derived in turn while the remaining independent variables are held constant. Residuals, i.e. differences between actual and calculated Y for each station, are plotted as deviations from the net regression lines to reveal, by successive approximations, the relationships between mean annual rainfall and each positional and site factor. The method is useful as it facilitates comparison of factors which jointly determine the mean annual rainfall at a point. A study of the southern Cape Province of the Republic of South Africa proved continentality to be a dominant influence, the hyperbolic net regression curve showing that mean annual rainfall diminishes rapidly with increasing distance from the sea up to about 100 miles inland whereafter the further decrease is negligible. Altitude emerged as another important control for, independently of its relationship to other factors, mean annual rainfall increases by about 5" per 1,000', but not regularly, the increase being more rapid from 0-1,000', and again between about 2,500' and 4,000' than

for other altitudinal increments; this may be due jointly to the general increase in altitude inland and to a topography comprising a series of mountain ranges parallel to the coast separated by intermontane valleys. The effect of aspect was not so definite but, other site factors being equal, mean annual rainfall on a southerly slope is about 7" more than on a northern slope. Whereas an increasing tendency from west to east was discernible on the mean annual rainfall map, this was scarcely apparant in the net regression curve of mean annual rainfall on longitude, which fluctuated about the horizontal; it was concluded that the apparent easterly increase was in fact a manifestation of interactions between other factors such as altitude and continentality.

d. While an estimate of mean annual runoff from an ungauged catchment can generally be hazarded from an indirect determination of mean annual rainfall (see sub-paragraph c), nothing is revealed of flow fluctuations occasioned by seasonal and annual rainfall variability. One promising solution lies in extracting maximum information from existing rainfall records and in mapping, even in a generalised manner, certain key parameters whose values for any catchment devoid of raingauges may then be read off the maps. Using these data it will often prove possible to construct an annual rainfall record (of any length) which is entirely synthetic but which possesses a frequency distribution and variability appropriate to the locality (3). The basic assumption is that in most regions the frequency of annual rainfall corresponds to a modified "normal distribution". A true "normal distribution" curve is completely defined by the mean of the items and by standard deviation. In the case of annual rainfall, modifications may be caused by skewness and kurtosis, both of which are easily determined. Having used this modified "normal distribution" to compute a synthetic annual rainfall record with a frequency distribution determined from the 4 parameters aforementioned, it remains to arrange the items in appropriate sequence. From probability theory it is easy to calculate the number of runs of wet and dry years, of various lengths which should occur in a random series, and should the locality be characterised by significant "persistence" in annual rainfall or alternatively by a significantly high proportion of short wet or dry spells (4), the distribution of items within the synthetic annual rainfall record can be modified accordingly. As a final refinement the annual record can be broken down into smaller components. Undoubtedly far more information can be extracted from long-term rainfall records than is generally attempted and even if the station network is comparatively sparse the mapped parameters can be widely useful.

3. ESTIMATION OF RUNOFF FROM RAINFALL DATA (J.P.Kriel and K.J.Harvey)

(a) Mean annual runoff figures are very useful as a first step in assessing the potential value of surface water resources. Once they have been established comparisons with the characteristics of similar gauged catchments can supply the information on which to base actual dependable yields.

(b) In 1950 a study of the available streamflow records and rainfall data of the Republic of South Africa was made (5). It was found that the relationship between mean annual runoff and average annual rainfall could be reasonably well represented by the expression $r = KR^n$ where "r" is the Mean Annual Runoff (M.A.R.), "K" a constant, "R" the average annual rainfall, and "n" an exponent which varied between 2 and 5. In view of the lack of data on very small catchments where rainfall is uniform, and the short-term records of which use had to be made in some cases, it was decided to adopt a value of n = 3 for further study of the mean annual runoff for the country as a whole.

(c) The first step in this study was to use all available rainfall records to draw an isohyetal map of rainfall normals on maps which also showed the topography of the country to a scale of 1 : 500,000.
(d) The next step was to compare the M.A.R. per unit area for different catchments with the rainfalls on those catchments. Records from 222 river gauging stations were used. Allowance was made for use of water upstream of the gauging stations. In view of the fact that it had been found that runoff varied approximately as the cube of the rainfall, the weighted average rainfall for each catchment was calculated from the equation

$$R_m = \left(\frac{\Sigma aR^3}{\Sigma a}\right)^{\frac{1}{3}}$$

where R is the average rainfall for each incremental area "a" of the catchment.
(e) The unit M.A.R.'s (i.e. M.A.R. per unit area) were then plotted against the corresponding weighted rainfall figures on logarithmic paper. After prolonged study the different points could be separated into similar groups and mean runoff-rainfall curves established for different regions of the country. Seven runoff zones were established in each of which runoff-rainfall relationships were similar, but these relationships varied appreciably between the different zones. The rainfalls required for different mean annual runoff figures were established and the variation between the different zones is illustrated in Table I, which gives some of the values determined.

TABLE I

Relationship between Mean annual runoff and rainfall for different zones in the Republic of South Africa.

Mean annual runoff, in acre-feet per square mile	Average annual rainfall in inches required to produce the given M.A.R. in each zone.						
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Zone 6	Zone 7
10	20.0	16.5	-	12.0	10.0	8.7	6.5
100	29.2	27.0	24.8	22.5	20.5	18.3	15.7
1,000	52.5	48.5	47.5	46.0	44.0	42.0	-

(f) By making use of the derived relationships and the rainfall map referred to in subparagraph 3(c), it was possible to plot equal runoff lines and thus to prepare a map showing the mean annual runoff for the Republic of South Africa. Experience since the map was prepared has shown up some weaknesses in it, but in general it has proved to be very useful in making estimates of water resources. Records for a further 15 years are now available, but estimates based on the map are usually well within 20% of runoffs obtained by making use of all the present data. In due course anomalies will be eliminated as more data become available.
(g) As implied in subparagraph 3(a) the determination of mean annual runoff is but a first step in assessing the water resources of a river. Draft curves derived from similar gauged catchments are used to determine assured yields, all curves being plotted in terms of the M.A.R. In various cases efforts have been made to extend records by means of annual or shorter term rainfall-runoff relationships, and although some improvement in accuracy has been obtained, the scatter of results for South African conditions is generally such that greater reliance cannot be placed on such estimates.
4. Sections 2 and 3 of this paper have dealt with methods for

assessing certain aspects of the water resources of rivers where inadequate records exist. Sections 5 and 6, on the other hand, deal with methods to improve the accuracy of data collection.

4. A GAUGING DEVICE FOR SEDIMENT-LADEN STREAMS (A.L.Rabie and R. de Villiers).

a. Sediment-laden streams tend to deposit their load upstream of gauging weirs built across them. This may seriously impair the accuracy of measurement as scour with subsequent re-deposition may take place, leaving no obvious evidence of conditions during a series of floods, so that actual bed levels and cross-sectional areas which may affect flow conditions are unknown. In addition, plant growth tends to establish itself in the saturated sediment deposits, thereby furthering the permanent build-up of sediment layers and clogging up the approaches above weirs, requiring costly maintenance. The high and variable velocity of approach above weirs which is caused by sediment deposits and the resultant vegetation build-up prevents accurate calculation of discharge tables for weirs for which stable stilling ponds or approach channels are required.

b. It is most desirable that the conditions upstream of a weir should be stabilised, that flood debris should pass the structure and that sediment and vegetation build-up be kept to a minimum. At the same time, in order to avoid the necessity of recording readings at different points, the discharge depth should be measured at a single point upstream of the entire measuring structure. A rectangular flume with a level bed, incorporated as a low notch to pass sediment at a gauging weir, was developed with these objects in view. One or more of these flumes with a width of 15 to 20 feet each are flanked by weir notches with crests at considerably higher levels.

c. A model flume with sides adjustable from 1 to 8 feet wide and 5 feet long was used to determine the necessary entrance conditions for smooth flow through the structure. The standard adopted as necessary for stable discharge relationships was that no shock waves should be formed in the flume with free flow. Initially the model had a right-angled entrance. The paths of water filaments flowing round these corners were determined for various depths of flow and flume widths. A study of the curves of these paths showed that they could all be related to a standard curve having two radii dependent on the relationship between the flume width and the water depth. Entrance transitions of these shapes were fitted a and further experiments carried out to determine the limits within which each set of transitions would be effective. It was found that smaller radii than those determined from the initial curves were satisfactory for equivalent depths of flow through the flume, but that there is a definite relationship between the entrance radii, the flume width, and the depth up to which smooth flow can be maintained through the flume.

d. The length of the flume excluding entrance transition, should be such that stable flow conditions are obtained within the flume. The experiments indicated that this required that the uniform rectangular portion of the flume should have a length of 4H, where H is the head as measured upstream of the entrance.

e. The shape of flume finally adopted is shown in FIGURE I. In FIGURE II is shown a dimensionless plot connecting the flume entrance radius R, the flume width B and the upstream depth H above the flume sill level. The curves indicate the zone for which satisfactory smooth flow conditions can be expected, a transition zone in which flow conditions are uncertain and a zone in which shock waves are likely to occur. With further study it is hoped to reduce the zone of uncertainty but the curves can be used for design as they are.

f. Flumes were calibrated by means of measurements on the model as well as by means of full-scale field measurements. It was found that downe stream water depths up to about 0.6H above the flume bed did not affect discharges. The flume can, however also be used as a measuring device with higher submergenves.
g. One installation with two flumes has been in operation in a sandy river for nearly two years, and so far the flumes have been effective in preventing accumulations of sediment upstream of the weir. Previously sand was deposited up to the weir crest.

5. AN IMPROVED METHOD FOR CALCULATING FLOOD PEAK DISCHARGES FROM SLOPE-AREA MEASUREMENTS (J.P.Kriel).

a. Flood peaks often cannot be gauged directly because of the short duration and irregular occurrence of spates. Frequently the best that can be done is to obtain levels from flood marks after the event, but flood peak calculations based on such data are subject to large errors, particularly due to the arbitrary choice of a roughness coefficient to be used in the calculations. In view of these considerations a method for calculating the flood peaks from water surface level data and river cross-sections has been developed which eliminates the arbitrary selection of the roughness coefficient. For applying this method at least three, but preferably four or more cross-sections with flood levels at them must be available, the friction coefficient over the reach of the river considered must be sensibly constant and the cross-sectional areas of the water prism must not all be equal.
b. By making use of the Manning formula for friction loss in open channels and the Bernouilli theorem, the discharge flowing between sections 1 and 2 in a river channel can be expressed by the equation

$$Q^2 = \frac{Z_1 - Z_2}{\frac{n^2}{8.833}\left(\frac{1}{A_1} + \frac{1}{A_2}\right)^2 \frac{L}{(R_m)^{4/3}} + \frac{a}{2g}\left(\frac{1}{A_2^2} - \frac{1}{A_1^2}\right) + \frac{a\,k}{2g}\left(\frac{1}{A_1^2} - \frac{1}{A_2^2}\right)}$$

where Q = discharge in cusecs
n = roughness coefficient in Manning formula $V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$
Z_1 & Z_2 = Water surface levels at sections 1 and 2 respectively.
A_1 & A_2 = Cross-sectional areas of water prisms at 1 and 2 respectively.
L = Length of river channel between sections 1 and 2.
R_m = Mean hydraulic radius for two sections.
a = Velocity distribution coefficient for kinetic energy. For natural channels this probably varies between 1.1 and 1.5(6).
k = Turbulence loss coefficient. For accelerating flow this may be taken as zero, but for expanding flow(i.e. velocity decreasing downstream) a value of 0.25 would probably be applicable where the enlargement of channel is gradual.

For a given reach of river in which flood levels and the cross-sections have been surveyed it will be noted that there are two unknowns in the equation, viz. the discharge "Q" and the value of the Manning coefficient "n", if it is assumed that the coefficients "a" and "k" can be determined with sufficient accuracy. Normally a is taken as unity in river hydraulics calculations, but it has been found that its correct value is of some importance for flood peak calculations. If the data for three cross-sections were known, a similar equation could be written between

sections 2 and 3, and with two equations it should be possible to solve for the two unknowns Q and n. Usually this is not possible however, for in picking up flood levels it is never possible to obtain exact water surface elevations. If exact water surface elevations are not used in the equations, the solution of them will yield imaginary values. Hence some adjustment of water levels to comply with the energy relationships on which the equations are based is necessary.

c. The procedure followed to overcome the difficulties encountered in solving the equation is as follows :

(i) Determine the most probable water levels at the sections and calculate the water areas and hydraulic radii.

(ii) By making use of equation calculate a curve of Q against n for each pair of sections such as 1 and 2, 2 and 3, as well as 3 and 4 if available, as shown in FIGURE III. If all water levels are correct these curves should intersect in one point. It will be seen from FIGURE I however, that the curves give three different intersections, and hence three different values of Q and n.

(iii) Decide on the two most reliable water levels at sections. Usually these can be taken at the first and last sections in the reach as this will yield the lowest probable percentage error due to the large overall drop in water surface. By assuming a mean value of Q and n as indicated by the intersections in FIGURE I, calculate the values $Z_1 - Z_2$, $Z_2 - Z_3$ and $Z_3 - Z_4$. The sum of these three calculated values should then equal $Z_1 - Z_4$ if Z_1 and Z_4 are accepted as correct. In general the sum will differ from the correct value, and the individual terms must be adjusted proportionally to yield adjusted values of Z_2 and Z_3.

(iv) Revised cross-sectional areas and hydraulic radii are then determined to correspond with the adjusted values Z_2^1 and Z_3^1 and the procedure outlined in subparagraphs (c)(ii) and (iii) repeated. The result of such an adjustment is shown in FIGURE IV, where it will be seen that the intersections have converged considerably. Systematic tabular calculations greatly simplify the method.

v. In many cases further adjustments will be necessary before the curves will intersect in one point. In the case illustrated here the values finally found were Q = 34,000 cusecs and n = .040.

(d) In a number of analyses to date it has been found that the effect of "k" on the discharge Q is usually small, but that the effect of variation in the kinetic velocity distribution coefficient α may be as large as 25%. Studies are planned to assist in assessing the value of this coefficient with greater accuracy.

(e) A complete discussion of the application of this method is not possible here. It has been expanded and applied to take account of composite channels where rivers overflow their banks. It has proved to be very useful to obtain reasonably satisfactory flood peak estimates where no other means could be used. In three cases where the results could be checked the figures obtained from rated stations were within 5% of the calculated values for two of the rivers and within 20% for the third. Even the worst case is equivalent to guessing the value of "n" within 20%, which usually is not practicable.

6. THE DETERMINATION OF THE MAGNITUDE-FREQUENCY RELATIONSHIP OF ANNUAL FLOOD PEAKS IN SOUTH AFRICA.(D.F. Roberts).

(a) While the use of unit hydrographs is advisable for important structures flood peak probability data are usually of great assistance. An empirical study was recently made to find a simple method of plotting the data on semi-logarithmic paper so that the mean line representing the magnitude-frequency relationship could be extended to estimate the

magnitude of a peak with a return period greater than the length of the record.

(b) The problem was to find some function of "n" the number of observations, and of "m" the rank of the m^{th} largest observation (the observations being arranged in descending order of magnitude) so that when plotted the points would be scattered about a straight line which could be fitted and extended mathematically with confidence.

(c) It was realised that individual records could not be used to determine this function since they are only small samples from the large population of flood peaks. It was therefore decided to combine twenty records of thirty five years duration, by standardising the observations and calculating the mean of each group of twenty standardised values corresponding to each value of "m". (Note that the standardised value Z_m is equal to $\frac{Xm - \bar{X}}{Sx}$ where Xm is the m^{th} largest flood peak and $\bar{X}$ and Sx are the mean and standard deviation of the n observed annual peaks).

d. These mean values of Zm form a composite sample of thirty five terms which provide a very good picture of the magnitude-frequency distribution of the parent population.

(e). The composite sample did not produce a straight line when plotted according to the "California Method" (7) in which the plotting position is represented by $T = {}^{n}/_{m}$, nor was the "Geological Survey Method" (8) where $T = \frac{n+1}{m}$ any better, while the "Gumbel Method" (9) proved to be the poorest, so that none of the methods provided a straight line that could be extended with accuracy. A straight line was produced by using the function $T = \frac{n+1}{m} + c$, the best value of c (=0.84) being found by trial and error, first graphically and then mathematically.

(f) The "Hazen function" ($T = \frac{zn}{zm-1}$) (7) produced a still better fit in that the standard error of estimate was lower and the points better distributed about the straight line, the regression equation being $Zm = 1.014 \log e \left(\frac{zn}{zm-1}\right) - 1.004 \pm 0.28$ which for the maximum peak, (m = 1.0) leads to $\frac{Xn - \bar{X}}{Sx} = 1.014 \log e\,(SN) - 1.004 \pm .028$ where Xn is the most probable maximum flood peak in a period of N years.

(g) This equation is theoretically applicable only when $\bar{X}$ and Sx are the mean and standard deviation of the observations in a thirty five year record. For other lengths of record the values of the coefficients would vary but the change would be negligible compared with the average standard error of the mean which in South Africa varies by ten to twenty percent. In view of this variation the following approximations are used without any serious error.

a. When the return period N is equal to or less than n, since

$$N = {}^{n}/_{m}\,, \quad XN = \bar{X} + Sx\left(\log e \frac{N}{1 - \frac{N}{zn}} - 1\right)$$

b. When the return period N is equal to or greater than n

$$XN = \bar{X} + Sx\,(\log e\, 2N - 1)\ldots$$

(h) Before calculating the mean and standard deviation of a record the maximum peak should be carefully scrutinised and if it appears to be a misfit its value should be adjusted to bring it into line with the rest of the data.

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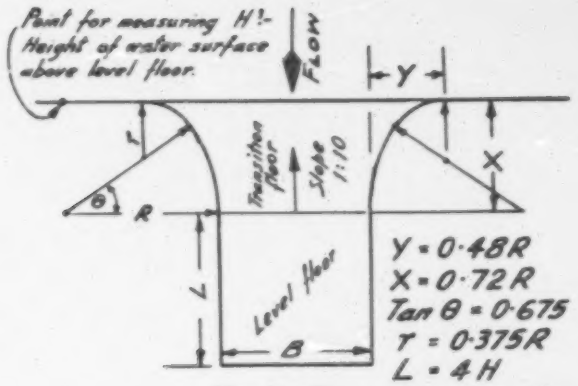

FIG. I. Plan of rectangular flume used in weirs in Silt-laden rivers.

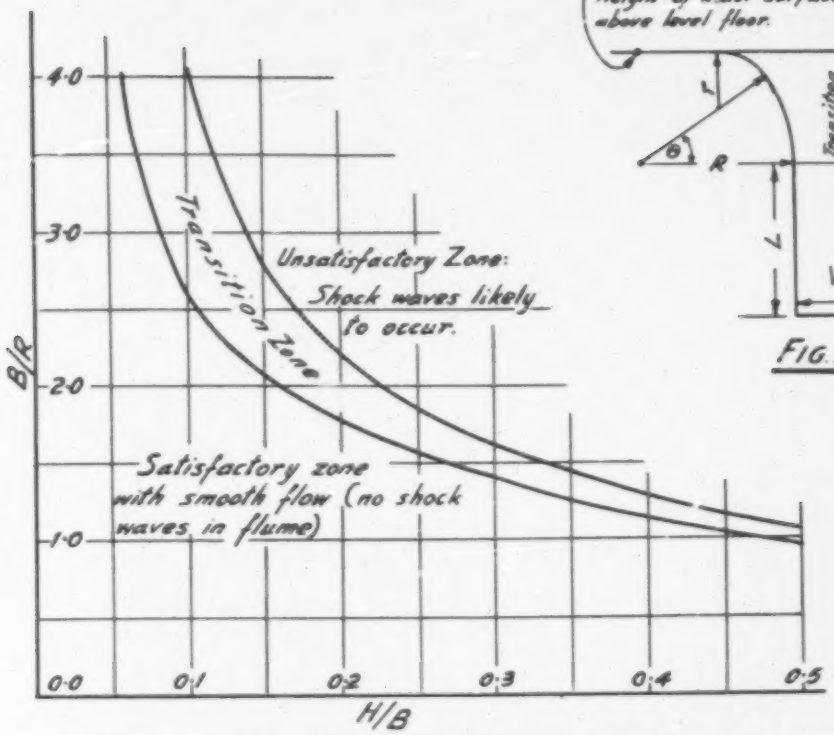

FIG. II Curves for designing entrance to rectangular flume shown in Figure I.

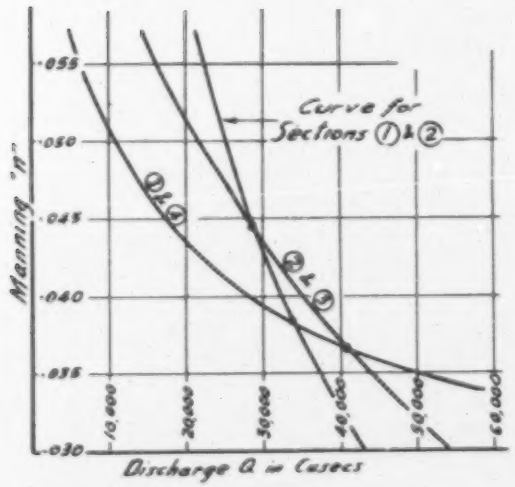

FIGURE III. Initial curves.

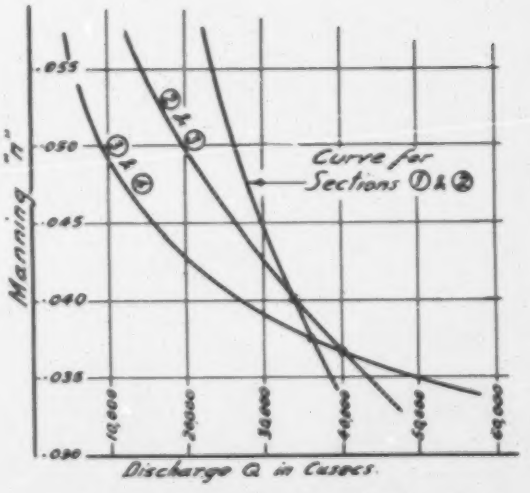

FIGURE IV. Curves after first adjustment.

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